INVESTIGATION OF THE MEPDG FOR USE IN OKLAHOMA CONCRETE PAVEMENTS

By

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FOR USE IN OKLAHOMA CONCRETE

PAVEMENTS

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CHAPTER I

THESIS INTRODUCTION

Construction of pavements using concrete started long ago. The first concrete pavement was constructed around 1913 in Pine Bluff, Arkansas in United State. Significant construction and design developments in recent years made paving faster and easier. This has allowed concrete pavements to be used in a larger number of areas. Development in pavement design and construction are still evolving with much more focus on life cycle costs.

The design of pavements is a very complex task as there are numerous variables involved. These variables have different levels of impact on pavements and change according to the details of the project. Previously we have used the AASHTO method (Highway Research Board, 1962) to design concrete pavements. This method relies on the empirical formulas based on performance of pavements. These empirical formulas were derived by AASHO road tests (Highway Research Board, 1962) in which both flexible and rigid pavements were considered. The test consisted of varying truck loads on different pavement thicknesses and design types. The performances of the pavements were monitored continuously for two years and the data was collected. Equations were then fit to the data and used as empirical design formulas.

While the AASHTO design guide has served pavement designers for over 30 years the following criticisms have been made (ARA 2004):

1) Modern traffic levels have increased by 10 to 20 times the levels since the time of the AASHO road test. Because only a limited amount of data could be obtained from the original test, extrapolation of the damage observed in the AASHO road test was needed to determine the long term performance of the pavements. While some extrapolation was deemed reasonable to determine the performance of pavements in the 1950s; however, this extrapolation would need to be taken to the extreme to meet modern traffic levels.

2) Environmental loading is thought to be an important component in the design of concrete pavements. Since the AASHO road test was only limited to pavements in Ottawa, Illinois and to a short period of little more than 2 years this key component cannot be modeled.

3) A limited number of construction materials were used in the construction of the test track. For example only one type of hot mix asphalt subgrade and only one concrete mixture was used.

4) The vehicle weights used for the test are now out dated.

5) The drainage system for the pavement was not been considered in the test.

6) Pavement rehabilitation procedures were not considered by AASHTO design guide.

7) Derived equations are complex and totally empirical and so they are not intuitive.

8) Limited guidelines for Continuously Reinforced Concrete Pavement (CRCP).

9) There is no information collected on pavement failure mechanism, just the “loss in service”.

Engineers needed a better approach for the design of concrete pavements. These efforts were started in 1996 as NCHRP 1-37 “Development of the 2002 Guide for Design of New and Rehabilitated Pavement Structures”.

A project design guide was developed which is based on the state of the art mechanistic and empirical research over pavements. It was named the Mechanistic and Empirical Design Guide (MEPDG). This guide uses mechanistic models to highlight which variables appear to be the most important. Then the design methodologies were “tuned” by using empirical data of field measurements to examine how they perform. The Oklahoma Department of Transportation (ODOT) wants to implement MEPDG for design of rigid pavements. The MEPDG has a number of inputs relating to climate, traffic, construction and design. Most of the inputs have some kind of default values or range. Therefore it was decided to find the most sensitive input parameters which will have maximum effect on the pavement thickness design. The effort to characterize the sensitivity of these variables and to determine values for Oklahoma concrete mixtures is the focus of this thesis.

1.0 Overview of Thesis

This thesis is structured in two major efforts. The sensitivity analysis of the MEPDG is primarily covered in Chapter 2 “Sensitive analysis of the rigid pavement design with the Mechanistic empirical pavement design guide.” The determination of shrinkage, compression, and flexure values for concrete pavement mixtures were determined in

Chapter 3 “Compressive, flexural and shrinkage testing for Oklahoma pavement mixtures.”

Finally, a summary chapter has been presented in chapter 4.

CHAPTER II

SENSITIVITYANALYSIS FOR THE DESIGN OF RIGID PAVEMENTS WITH THE MECHANISTIC EMPERICAL PAVEMENT DESIGN GUIDE

2.0 Overview

This focus of this chapter is on understanding the MEPDG and its parameters which can be sensitive to the design of Oklahoma pavements. The design software that forms the MEPDG was developed by Applied Research Associates (ARA) through several projects from the National Cooperative Highway Research Program (NCHRP). The goal of the software is to provide a new design methodology for concrete and asphalt pavements based on the latest failure mechanisms in combination with empirical data from the performance of pavements in the field.

Before the release of the MEPDG it was common for designers to use a version of the AASHTO design guide. This design method has seen several different iterations that vary from hand methods that use nomographs to simple software interfaces. The AASHTO design guide is based on empirical performance of several miles of test track in Ottawa, Illinois from 1958 to 1960. This testing is commonly called the AASHO road test. For this testing, pavements were continuously loaded with trucks over a period of a little more than two years.

Due to a number of limitations in AASHTO method as discussed in chapter 1 it was decided to replace it with MEPDG.

The MEPDG was designed to improve on the previous AASHTO design methodologies. The creators of the MEPDG feel that the short comings can be overcome if one is able to fundamentally define the performance of a pavement through the use of the latest mathematical models that describe behavior in combination with the measurement of the actual performance of pavements with a significant number of differences in climate, loading, and construction materials. These empirical observations are imperative to help the mathematical expressions to become meaningful and useful.

2.1 Review of the inputs to the MEPDG and the past review of their significance on the final design values

2.1.1 Variables in the MEPDG

The MEPDG software allows the user to change over 150 variables that impact the performance of the pavement. These variables have been grouped by category including: climate, traffic, pavement layers and material properties.

2.1.1. 1. Significance of Variables

While it is helpful to provide designers with a large number of variables that they are able to control in order to tailor their pavement design, this can also be a challenge as the number of variables can be overwhelming to specify, measure or control. Instead it would be more useful for designers to understand which variables have the largest impact on their designs or are the most significant. Other researchers have attempted to determine which variables have the biggest impact on the results of the MEPDG (Zaghloul et. al 2006, Kannekanti 2006, Harvey 2006, Mallela et. al 2005, Harrigan and Nov 2002).

While the previous work is useful some common difficulties were found including:

- No information about the MEPDG software version that was used for the analysis;
- Little information is given about the metrics used to determine if an input was significant;
- No constant metric was used across investigations to compare results, and;
- Lack of detail of the range of values used in the analysis.

Because of these inconsistencies and the desires for ODOT to implement the MEPDG software, it was decided to perform a new sensitivity analysis on the MEPDG. After reviewing the list of possible variables that can be modified, and through discussions with ODOT, a list of variables were chosen to be investigated that were deemed reasonable to be able to control in the field. A summary of these variables is shown in Table 2.1. These variables were investigated to quantify if the results of the MEPDG were sensitive to these parameters.

2.2 Sensitivity Analysis

This sensitivity analysis was completed between March and August of 2009 with version 1.0 of the software that was obtained from the MEPDG website, www.trb.org/mepdg/software. Along with this software hourly climatic data files from version 0.910 were also downloaded. It should be noted that the results from the

MEPDG may not be the same if a different version of the software or if a different set of climatic data was used in the analysis.

2.2.1 Analysis Methods

It was decided that all of the comparisons of these previously mentioned variables should be done on a common metric that was easily accessible to a pavement design engineer. One easily recognizable variable to design engineers is the required pavement thickness. Unfortunately, the current version of the MEPDG does not provide the user with a satisfactory pavement thickness for the variables presented. Instead, it analyzes the pavement design with the variables used and will report if the pavement is adequate. Therefore, to investigate the sensitivity of these different variables on the required pavement thickness, it was decided to start with a pavement design that was representative of an ODOT pavement and find the Annual Average Daily Truck Traffic (AADTT) that made it just adequate. A variable was then modified and the pavement was analyzed to see if the section was adequate. If the pavement was not adequate then the pavement thickness was increased until the design was reported as adequate. If the pavement performance was increased by the change in the variable then the pavement thickness was decreased to find the thickness that just allowed it to be adequate. By using this technique then it was possible to find how a single variable impacted the thickness design for a pavement.

Both continuously reinforced concrete pavement (CRCP) and jointed plain concrete pavement (JPCP) were considered for analysis. For these pavements the edge support was assumed to be a tied PCC shoulder and PCC-base interface was kept as full friction contact. The material properties of the asphalt, used as a bond breaker, was chosen to meet ODOT standards and not varied. The default parameters from the MEPDG were used unless noted in Table 2.2.

Table 2.1 – A summary of the baseline values for Oklahoma pavements.

design life	20 years
cement	600 lbs of type I
concrete flexural strength	750 psi / 690 psi
Curing	curing compound
shoulder	Tied
JCP dowel diameter	1.5 inches
CRCP reinf. Ratio	0.70%
location	Stillwater
pavement opening	Fall
base layers	4 inches asphalt 8 inches chemically stabilized base
Subgrade	8000 psi resilient modulus

In order to find the AADTT for the various thickness of JPCP and CRCP that caused failure of the pavement section, a pavement section was created with the previously mentioned baseline parameters and the AADTT values were increased until the pavement was found to just be unsatisfactory. This allowed the limiting AADTT to be determined for the chosen design variables and thickness. Above procedure is shown as a flowchart in the Figure 2.1 and 2.2. A summary of the results is shown in Figure 2.3.

Fig 2.1 Flowchart showing steps to find out critical AADTT

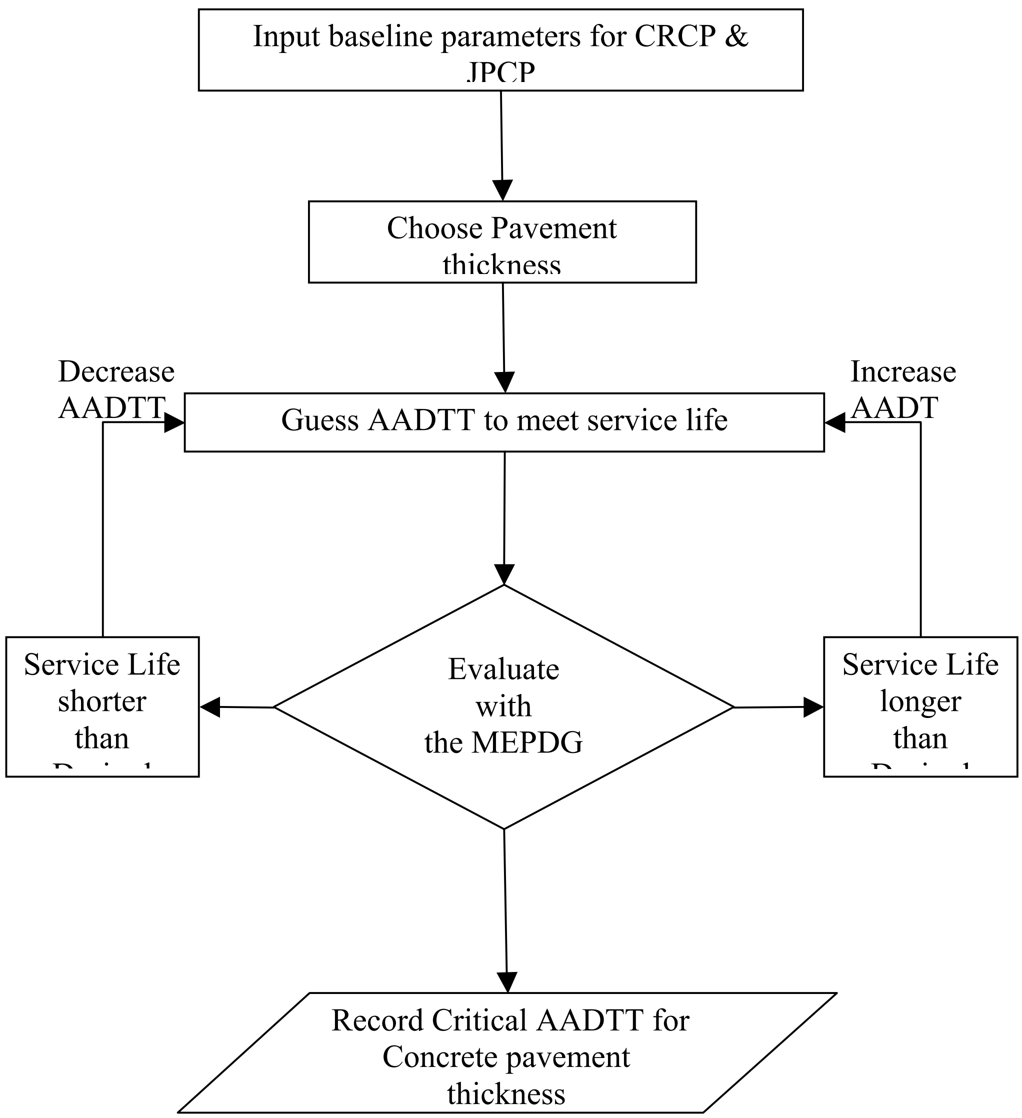
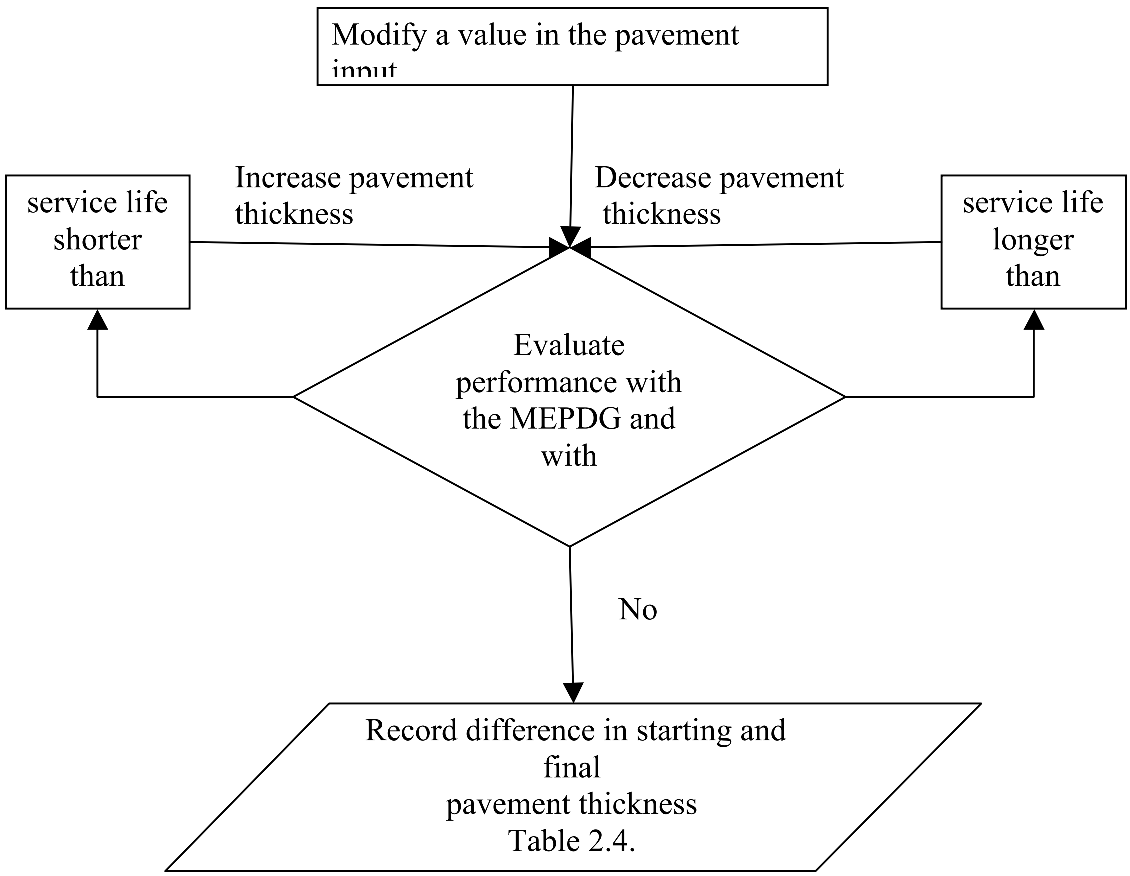


Fig 2.2 Flowchart showing steps to find out the Thickness change



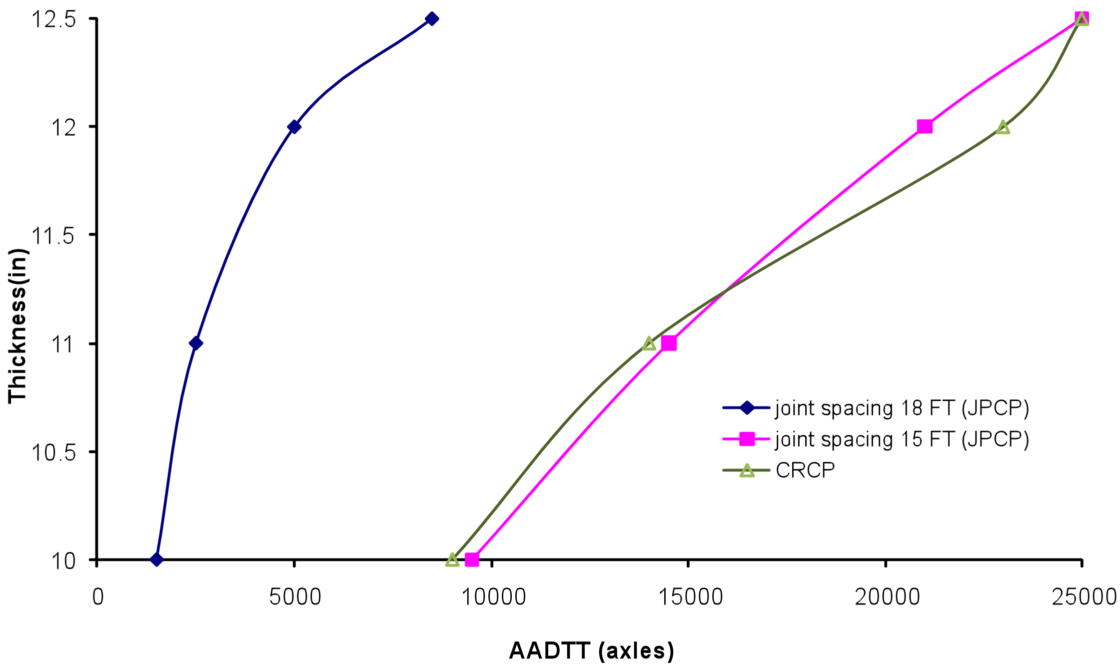


Figure 2.3 – A plot of the required design thickness for JPCP and CRCP with different AADTTs.

After a baseline AADTT was found for a pavement then the sensitivity analysis could begin. Next a variable from Table 2.1 was modified in the design and the pavement thickness was also adjusted until the pavement was found to just be acceptable. This allowed the impact on the thickness to be determined for a single variable for a given set of parameters. The range that each variable was varied over is summarized in Table 2.2.

Table 2.2 - A summary of the variables and their ranges used in the sensitivity analysis.

CRCP and JPCP parameters are:

Parameter	Range
Pavement opening	fall, spring or summer
Co-efficient of Thermal Expansion (CTE)	3.5-8 * 10^{-6} / ^{0}F
cement type	type I or II
curing	compound or wet
compressive stress	3000-6000 psi
cementitious material content	400-800 lbs/cy
asphalt layer thickness	0 – 6 inches
cement fly ash layer thickness	0 - 8 inches
reinforcement ratio (CRCP)	0.5 - 1%
dowel diameter (in) (JPCP)	1 - 1.75 inches
unbound resilient modulus of the subgrade	3000 - 13000 psi
climate	Stillwater, Clinton, Lawton, McAlester Oklahoma city, Tulsa, Frederick

2.3 Results

The impact of a change in each variable is reported in terms of the change in the required pavement thickness in the MEPDG to make the section adequate. A summary table is reported in table 2.4.

For each case the change in the pavement thickness in inches is 1/2 inches. A plus was used for a required increase in thickness and a minus was used for a required decrease in thickness. In some cases there was no impact on the thickness and a "0" is reported. A letter is also reported next to each thickness change that designates the failure mode that governed for that analysis. All default values are indicated by an asterisk and values that were not expected were shown in bold. The default failure criteria established by the MEPDG was used in each analysis. These are summarized in Table 2.3.

Table 2.3 – A summary of the failure criteria used in the sensitivity analysis.

CRCP failure criteria	limit	reliability
terminal International roughness index (IRI)(in/mi)	172	90
CRCP Punchouts (per mi)	10	90
maximum CRCP crack width (in)	0.02	
minimum crack load transfer efficiency (LTE%)	75	
JPCP failure criteria		
terminal IRI (in/mi)	172	90
transverse cracking (% slab cracked)	15	90
mean joint faulting (in)	0.12	90

The failure criterion is explained below:

International roughness index (IRI) - It is the check for the smoothness of a pavement. It is defined as average rectified slope (ARS), which is a ratio of the accumulated suspension motion to the distance traveled obtained from a mathematical model of a standard quarter car traveling in a measured profile at a speed of 50 mph (80 Km/h). Its units are inches per mile.

CRCP punchout - A punchout is a major distress in CRCP. Failure starts with one or two transverse cracks. These cracks widen causing the slab to behave like a cantilever beam. Further repetitions of heavy loads causes fine longitudinal cracks in between the two transverse cracks. Eventually the transverse crack break down further, the steel ruptures, and pieces of concrete punch downward under the load. Punchouts are measured by counting the numbers that exist per mile.

Crack width - It plays an important role in load transfer and determining the amount of steel required in CRCP pavements. The load transfer efficiency decreases as the cracks get wider. This eventually leads to load related critical tensile stresses at the top of the slab, increasing the fatigue damage and eventually the development of longitudinal cracks and punchouts. Limiting mean crack width is taken as 0.02 inches at the depth of steel.

Load transfer efficiency (LTE) - It is the ability to transfer loads between the pavement slabs as well as between the cracks. LTE is important factor in controlling the punchout related longitudinal cracking. LTE is limited to 75%.

Transverse cracking - When the loading is near the longitudinal edge and in between the transverse joints, critical bending occurs at the bottom of the slab. This stress further gets amplified due to high positive temperature gradient. Repeated loading under those conditions results in fatigue damage and cracking in the bottom of the slab which eventually results in the transverse cracking. Transverse cracking is measured as percentage and is limited to 15% in MEPDG.

Mean joint faulting - Faulting is the difference in the elevation across the joints or cracks. As faulting is not even throughout, mean joint faulting is considered. Faulting is limited to 0.12 inches in MEPDG.

Reliability - In order to assure certainty in the design process for a given life reliability is considered. In the sensitivity analysis reliability is taken as 90 % for all the MEPDG runs.

The following example is used to illustrate the use of the table. If we have a 12 inches CRCP pavement that is adequate and we change the CTE value of the aggregate from 5.5×10^{-6} to 6.5×10^{-6} then the design thickness will have to be increased by 1.5 inches to 13.5 inches to make the pavement adequate for the same AADTT. The controlling failure mechanism will be cracking and inefficient load transfer. This technique is able to quantify the impact of a change in a given variable on the design thickness of the pavement.

Table 2.4 - Results from MEPDG sensitivity analysis.

material parameters

	CRCP				JPCP (spacing 18 FT)				JPCP (spacing 15 FT)			
parameters	12.5"	12"	11"	10"	12.5"	12"	11"	10"	12.5"	12"	11"	10"
cement type												
I*	0"L	0"P,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"T,J
II	-0.5"L	-0.5"P,L	- 0.5"P,L	0"L	0"T	0"T	0"T	0"T	0"J	0"J	-0.5"J	0"T,J
curing												
curing compound*	0"L	0"P,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"T,J
wet cure	-1" P,L	-0.5" L	-0.5"L	-0.5"P,L	0"T	0"T	0"T	0"T	0"J	0"J	-0.5"J	0"T,J
cement content (lbs/cy)												
500	-1"L	-0.5"L	-0.5"L	-0.5"P,L	0"T	0"T	0"T	0"T	0"J	0"J	-0.5"J	0"T,J
600*	0"L	0"C,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"J
700	**>+3"C,L**	**>+3"P,C,L**	**>+3"P,C,L**	**>+3"P,C,L**	+0.5"T	0"T	0"T	0"T	+1"J	+1"J	+0.5"J	+0.5"J
compressive strength (psi)												
3000	**0"P,L**	**0"P,C,L**	**0"I,P,L**	+0.5"P,L	+3.5"I,T	+3"I,T	+3"T	+3.5"T	+1.5"T	+1.5"T	+2"T	+1.5"I,T
4200	+0.5"L	+1"P,C,L	+0.5"P,C,L	+0.5"P,C,L	+1.5"I,T'	+1.5T	+1"T	+1.5"T	+0.5"J	0"J	0"T,J	+1"I,T
5000	0"C,P,L	+0.5"L	+0.5"C,L	+0.5"C,L	+0.5"T	+0.5"T	0"T	0"T	0"J	0"J	0"J	+0.5"T
6000	0"L	+0.5"L	0"P,L	+0.5"L	-0.5"T,J	-0.5"T	-1"T	-1"T	0"J	0"J	0"J	0"J
CTE ($1x10^{-6}/^{o}F$)												
4.5	-0.5"L	0"L	0"L	0"L	**-3"T**	**-2"T**	**-2.5"T**	**-2"T**	**>+3.5"J**	**-1.5"J**	**-3"J**	-1"I,T
5.5*	0"L	0"C,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"	0"J	0"J	0"J
6.5	**+2"C,L**	**+1.5"C,L**	**+1.5"C,L**	+1"C,L	**+3.5"T**	**+2.5"I,T,J**	**+2"T**	**+2.5"T**	**+3"I,J,T**	**+1.5"I,T,J**	**+2"J**	**+2.5"I,T,J**
resilient modulus (psi)												
3000	-0.5"L	0"L	0"L	0"L	0"J	**-1.5T**	**-1.5T**	**-0.5T**	+2J	+2J	+1"J	+1"J
5500	0"L	0"L	0"L	0"P,L	0"T	**-0.5"T**	**-0.5"T**	0"T	+0.5"J	+0.5"J	+0.5"J	+0.5"J
8000*	0"L	0"P,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"J
10500	0"L	+0.5"L	0" P,L	+0.5"L	**+0.5"T**	**+0.5"T**	0"T	0"T	0"J	0"J	-0.5"J	0"J
13000	0"L	+0.5"L	+0.5"L	+0.5"L	**+1T**	**+0.5"T**	**+0.5"T**	0"T	0"J	0"J	-0.5"J	+0.5"T

The required change in pavement thickness to insure comparable performance for the change in the variable. Positive values suggest an increase in thickness negative a decrea

* - values used to represent typical ODOT pavements, therefore they have no impact on the pavement thicknes

Bold values correspond to a result that was unexpected.

The controlling failure mode is given by the letters.

L - load transfer efficiency

P - punchouts

J - joint faulting

T - transverse cracking

I - IRI

C- cracking

design parameters

parameters	CRCP				JPCP (spacing 18 FT)				JPCP (spacing 15 FT)			
	12.5"	12"	11"	10"	12.5"	12"	11"	10"	12.5"	12"	11"	10"
pavement opening												
Summer	-0.5" L	0" L	0" L	0" L	+0.5" T	0"T	0"T	0"T	0"J	0"J	-0.5"J	0"I,T,J
Spring	**-2"L**	**-2"P,L**	**-1.5"PL**	**-1"P,L**	+0.5"T	0"T	0"T	0"T	-0.5"J	-0.5"J	**-1"J**	0"I,T,J
Fall*	0" L	0" P,L	0" P,L	0" P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"I,T,J
reinforcement steel (%)												
0.6	**>+3.5"P,C,L**	**+2P,C,L**	**>+3.5"P,C,L**	**>+3.5"P,C,L**	-	-	-	-	-	-	-	-
0.7	0" L	0" P,L	0" P,L	0" P,L	-	-	-	-	-	-	-	-
0.8	-1.5"L	-1"L	-0.5"L	-0.5"L	-	-	-	-	-	-	-	-
dowel diameter (in)												
1	-	-	-	-	**>+3.5"I,J**	**+3.5"I,J**	**+3"I,J**	**+2.5"I,J**	**>+3.5"I,J**	**>+3.5"I,J**	**>+3.5"I,J**	**>+3.5"I,J**
1.25	-	-	-	-	**>+3.5"I,J**	**+3.5"I,J**	**+2.5J**	0"T	**>+3.5"I,J**	**>+3.5"I,J**	**>+3.5"I,J**	**>+3.5"I,J**
1.5*	-	-	-	-	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"T,J
1.75	-	-	-	-	0"T	0"T	0"T	0"T	-0.5"J	-0.5"J	-1"J	0"J
asphalt layer thickness(in)												
0	0"L	+0.5"L	+0.5"L	0"L	**>-3.5"T**	**-2.5"T**	**>-3.5"T**	**>-3.5"T**	**-1"J**	**-2"J**	**-3"T**	**-1.5"T**
2	0"L	0"L	0"P,L	0"P,L	+1"T	+0.5"T	+0.5"T	+0.5"T	+1"J	+0.5J	0"J	+1"T
4*	0"L	0"P,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"I,T,J
lime cement fly ash stabilized layer (in)												
0	0"J	+0.5"P,L	0"P,L	+0.5"L	0"J	**-0.5"T**	**-0.5"T**	**-0.5"T**	+1.5J	+1"J	+0.5"J	+0.5"J
3	0"T	0"L	0"L	0"P,L	0"T	**-0.5"T**	**-0.5"T**	**-0.5"T**	+0.5"J	+0.5"J	**-0.5"J**	0"T
5	0"T	0"P,L	0"P,L	0"P,L	0"T	**-0.5"T**	**-0.5"T**	0"T	+1"J	+0.5"J	0"J	0"T,J
8*	0"T	0"P,L	0"P,L	0"P,L	0"T	0"T	0"T	0"T	0"J	0"J	0"J	0"T,J
Climate												
Clinton	**>+3.5"C,L**	**>+3.5"C,L**	**+3"C,L**	+1.5C,L	0"T	-0.5"T	0"T	-0.5"T	-0.5"J	-0.5"J	-1.5"T	0"T
Fedrick	+0.5"L	+0.5"L	+0.5"L	+0.5"L	+0.5"T	+0.5"T	+0.5"T	+0.5"T	-0.5"J	-1"J	-1"T	+0.5"T
Lawton	0"L	+0.5"L	+0.5"L	+0.5"L	+1"T	0"T	0"T	0"T	-1"J	-1"J	-1"T	+0.5"T
Tulsa	-0.5"L	0"L	0"L	0"L	-0.5"T	0"T	-0.5"T	0"T	-0.5"J	0"J	0"J	0"T,J
Stillwater*	0"L	0"L	0"L	0"L	0"T	0"T	0"T	0"T	0"T	0"T	0"T	0"T
McAlester	-0.5"L	0"L	0"L	0"L	-0.5"T	0"T	-0.5"T	0"T	0"J	-1"J	-1"T	0"T

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C- cracking

2.4 Discussion

From the results it is clear that some variables had a more significant impact than others. Furthermore, these variables often had different impacts on the different pavement types and thicknesses investigated. Before starting the discussion about various sensitive parameters; the impact of joint spacing on AADTT is considered. Looking at Fig. 2.1, it can be said that change in the joint spacing from 18 feet to 15 feet causes the AADTT to be increased by more than 300%. The main purpose of joint spacing is to control the cracking resulting from tensile and bending stresses caused by shrinkage, climate, and traffic loadings. It is understood that decreases in the length of joint spacing will increase the load carrying capacity, but the amount of increase in the AADTT according to MEPDG seems to be very high and may not be rational. Also notice that the performance of JPCP with a joint spacing of 15 feet has similar performance to CRCP with an equivalent depth. Again it is not clear if this is rational. A discussion for each one of the variables is provided along with a summary in Table 2.6.

The season that the pavement was opened to traffic had little impact on the JPCP. However the CRCP pavements that were opened to traffic in the Spring were able to be reduced in design thickness between 1inches and 2 inches. Selecting a season can have an effect on the "zero-stress" temperatures in the PCC at the construction. Increase and decrease in temperature with respect to zero-stress temperature can induce thermal stresses in the concrete. Chances of higher zero-stress temperatures are in summer or fall compared to spring. This can be the reason for reduction in the required thickness for CRCP pavements which were opened in spring. More work is needed to determine why this is and if it is rational.

The curing type had little impact on JPCP design thickness. However, it consistently impacted the design of CRCP by allowing for a decrease in design thickness of up to 1 inches. This size of impact makes this parameter significant for thicker CRCP. The primary aim of curing is to promote hydration near the surface of the concrete and to keep moisture and temperature profiles constant during curing. It is seen from the experiments that concrete temperatures during the first day of hardening are decisive for the thermal stresses and the cracks in concrete. Curing done with curing compounds or foil sheets induces more thermal stresses and high hardening temperatures, which in turn increases the risk of cracking. Whereas in wet curing cooling is due to evaporation ,so low thermal stresses as well as low hardening temperature is seen which in turn also reduces the risk of cracking and increases the durability.

The coefficient of thermal expansion (CTE) is defined as the change in unit length per degree rise in temperature. The CTE can contribute to pavement curling and stresses in the pavement from subgrade friction. Both of these can cause slab cracking which further affect the LTE resulting into CRCP punchouts and increase stresses in JPCP from combined stresses of curling and traffic loading. The CTE was a variable that consistently had a significant impact on the design thickness of both CRCP and JPCP. When a CTE value of 6.5 was used instead of 5.5, there was a significant increase in pavement design thickness. A lower CTE value of 4.5 has very little impact on the design thickness of CRCP. However, this same change had a significant impact on JPCP. This leads to changes of over 3 inches in some cases and was the most significant variable investigated. At this time it is unclear if this result is reasonable in terms of numbers but the behavior seems to be more or less justifiable. The cement type used allowed a 0.5 inches reduction of the design thickness with CRCP but has no impact on JPCP. This was due to an assumed difference in the shrinkage of concrete made with different types of cement.

Compressive stress has more impact on JPCP compared to CRCP. Lower compressive stress of 3000 psi caused significantly higher pavement thicknesses to be required for JPCP. Higher compressive strengths allowed for a reduction in thickness. This somewhat makes sense as the tensile strain capacity should be higher and so therefore the resistance to cracking. A small impact was made on the design thickness for CRCP whether the compressive strength was higher or low. There appears to be some error in the analysis of CRCP with 3000 psi compressive strength. These pavements actually showed less of an impact than the same sections with 4200 psi. This is unexpected.

When the cementitious material content was increased this resulted in higher shrinkage strains in the pavement. Drying shrinkage develops cracks in CRCP and reduces the load transfer efficiency. In JPCP drying shrinkage causes slab warping and further faulting because of differential shrinkage due to variation in moisture conditions throughout the thickness. The MEPDG suggests that this variable has a higher impact on CRCP pavements than JPCP. Higher cementitious content caused substantial increases in the design thickness for CRCP. This increase was much more substantial than in JPCP. For the CRCP pavements investigated, a lower cementitious content allowed for a reduction in thickness.

Base layers are important component of a pavement unit. Faulty or under designed base layers can affect crack spacing pattern, slab support and loss of support, punchouts, smoothness, and construction costs. As the thickness of the asphalt layer was reduced for the JPCP from the 4 inches default value, the required thickness of the pavement increased, which was an expected change. However, when this layer was removed from the analysis results show that a decrease in the required pavement thickness is allowed. This behavior is not expected. The asphalt layer thickness showed very little impact on the CRCP design thickness. This suggests that the base material has little impact on the required design thickness for CRCP.

As the lime cement fly ash layer thickness was decreased in the design it showed a reduction in the pavement thickness for JPCP with 18 feet spacing but for JPCP with 15 feet spacing it shows an increase in the thickness required. This doesn't seem to be logical as the expected result was either an increase or no change in thickness for 15 feet joint spacing with JPCP pavement. Again the reason for this behavior is not intuitive. There was not a significant impact of the lime cement fly ash layer thickness on the CRCP design thickness. This again suggests that the base material has little impact on the required design thickness for CRCP.

Changes in the stiffness of the unbound resilient modulus of the subgrade for JPCP with 18 feet spacing and 15 feet joint spacing showed an exact opposite response. The pavements with 18 feet joints suggested that as the stiffness of the unbound resilient modulus decreased that the pavement thickness could also decrease. Furthermore, when the unbound resilient modulus was increased the pavement thickness was required to be increased. While it is reasonable to assume that the stiffness of the base should have an impact on the design thickness of a pavement, the research team expected that the performance of the JPCP with 18 feet joint spacing would behave similarly to one with a

15 feet joint spacing. In the CRCP investigations the resilient modulus had almost no impact on the required design thickness of the pavement.

Dowel diameter is one of the governing factors affecting the concrete bearing stress and joint faulting in JPCP pavements. Therefore increase in the dowel diameter should reduce the distresses and the required design thickness Increase in the dowel diameter from 1.5 inches to 1.75 inches showed no effect on the thickness design for either JPCP sections investigated. However, a change in the dowel diameter from 1.5 inches to 1.25 inches leads to increase in pavement thickness designs of over 3.5 inches. It is unclear why such a small change in a variable can lead to such a significant change in thickness.

The reinforcement ratio for CRCP was another important design variable; it is obvious that increase in the reinforcement ratio will assure tight cracks over the design life. But reinforcement ratio for CRCP showed significant changes in the required design thickness for small changes in the value. For example a change from 0.7% to 0.6% required a thickness change in several cases of over 3.5 inches. This is a drastic change in the design thickness for a very small reduction in the amount of reinforcement. In turn an increase in the amount of reinforcement led to a small decrease in the required design thickness.

Several different cities within Oklahoma were chosen to evaluate how the different environments in the state impact the pavement design thickness. The majority of these cities had very little impact on the design thickness of the pavements investigated. However, for some reason the climate for the city of Clinton has a significant impact on the design thickness for thicker CRCP. But by looking at the table 2.5, it can be concluded that the climatic parameters for Clinton were not drastic, in fact the climatic values were almost in the same range as of the other cities. Therefore the impact of Clinton's climate on CRCP pavements is not justified. This behavior was only for CRCP as the JCPC were not significantly impacted by the environment in Clinton.

Table 2.5 – Comparison of climatic parameters of Clinton with other cities of Oklahoma.

Climatic parameters	Clinton	Frederick	Lawton	McAlester	Tulsa	Stillwater
Mean annual air temperature (ºF)	60.87	62.78	62.11	62.09	60.57	59.92
Mean annual rainfall (in)	20.28	20.26	26.28	30.95	38.89	29.31
Freezing index (ºF-days)	136.84	102.6	138.96	118.48	203.55	211.28
Average Annual Number of Freeze/Thaw Cycles	46	41	58	43	61	57

Table 2.6 – A summary of the impact on the thickness design requirement for each of the investigated variables.

	intensity of impact		
parameter	CRCP	JPCP 18 feet joints	JPCP 15 feet joints
cement type	low	none	low
curing compound	high	none	low
cement content	high	low	high
compressive strength	high	high	high
CTE	high	high	high
resilient modulus	low	high	high
pavement opening	high	low	high
reinforcement percentage	high	-	-
dowel diameter	-	high	high
asphalt thickness	low	high	high
thickness of stabilized layer	low	low	high
climate	high	high	high

none = no impact
low = 0.5 inches or less
high = greater than 0.5 inches

2.5 Conclusion

In this study a sensitivity analysis was completed that allows the user to quantitatively compare the impact of different variables on the design thickness in the MEPDG for CRCP and JPCP. The ability to quantify the impact of these different variables in this manner was not found in any previous publication or journal paper. While completing this sensitivity analysis several variables were found to make a much more significant impact than was expected. Also, several variables behaved in ways that were unexpected. No combination of variables was investigated beyond what is presented here and so care should be taken in applying any combinations of variables.

The research team feels that an owner should be careful in using the MEPDG for pavement design as the results of the analysis do not seem intuitive. Several small changes in input data required pavement thickness designs of significantly different values. Furthermore, the equations and processes used to calculate the performance of the pavements should be more transparent to the user of the software. This would make it easier for a designer to determine how the different models are being applied.

CHAPTER III

COMPRESSIVE FLEXURAL AND SHRINKAGE TESTING FOR OKLAHOMA PAVEMENT MIXTURES

3.1 Introduction

As a way to improve the input parameters in the MEPDG it was decided to obtain actual shrinkage and strength values for common concrete mixtures from the state of Oklahoma. Shrinkage is one of the important parameters in concrete in terms of serviceability. Shrinkage can lead to dimension changes of concrete and ultimately cracks when it is restrained. Concrete pavements are typically restrained by the support base. When a concrete pavement shrinks any restraint causes tensile stresses. When the tension stress exceeds the tensile strength of the concrete the concrete will crack. Reducing shrinkage can help in reducing cracks in the pavement. Amount of shrinkage depends upon various factors such as amount of free water in the concrete, cementitious material content, aggregate size, fineness of gel, and climatic conditions.

Shrinkage affects the load transfer efficiency in CRCP and cause warping of JPCP which further results in faulting.

Strength is another important parameter in concrete. Both the flexural and compressive strength of concrete is an indirect measurement of the tensile strength of the material. This is important as it allows an estimate of the strain that the concrete can undergo before fracturing. Typically the flexural strength is generally about 10 to 20 % of the compressive strength. The flexural strength of concrete according to MEPDG should be $9.5\sqrt{f'c}$.

Generally the strength of concrete is influenced by cement type, cement content, presence of admixtures, water to cementitious ratio, use of supplementary cementitious material, curing, age, test condition, method and equipment. Aggregate properties like type, maximum nominal size, gradation, particle shape and texture can also impact the strength.

3.2 Experimental Methods

3.2.1 Shrinkage

For this testing ASTM C 157/C-04 / AASHTO T 160 was used to evaluate the shrinkage potential of typical Oklahoma concrete pavement mixtures. However, as suggested in the MEPDG manual, the relative humidity used for testing was lowered to 40%. In this testing 13 different concrete mixtures were made. These mixtures were chosen based on a survey of typical Oklahoma pavement mixtures from contractors. The cements used for this testing were Holcim from Ada, Lafarge from Tulsa and Buzzi from Tulsa. All of these cements are ASTM C 150 type I/II cements. Four different Oklahoma fly-ashes were used. The fly-ashes used were Redrock from Ponca City, Oklaunion from Oklaunion, TX, Muskogee from Fort Gibson, OK, and GRDA from Chouteau, OK. A summary of these mixtures are given in Tables 3.1 and 3.2. For every concrete mixture three concrete prism and 12 cylinders were made for testing out of which three cylinders were tested on the 7th and three more were tested on 28th day. The rest of the cylinders were reserved for future coefficient of thermal expansion testing.

3.2.2 Flexure

For Flexure testing ASTM C 78-08 / AASHTO T97 was used to evaluate six different concrete mixtures. The same cement and fly ash mixtures were investigated as the shrinkage testing. Details regarding these mixtures are in Tables 3.1.and 3.3.
For every concrete mix 12 concrete beams and 12 cylinders were made for testing. Cylinders were tested at 3, 7, 28 and 90 days.

3.3 Mixture Proportions

The amount of saturated surface dry aggregates and cementious material used for making the mixture per cubic yard and other fresh concrete properties are listed below in table 3.1.
Mixture 1 to 3 is mixed without fly ash whereas mixture 4 to 13 contains fly ash. Mixture 11, 12 used different sacks of cementitious materials i.e. 6.5 and 5.5 sacks respectively whereas all other mixtures are 6 sacks of cementitious materials.

Table 3.1. Mixture Designs Quantities for Shrinkage and Flexure

Mixture #	Cement Type	Fly-Ash Type	Cement (lb/yd^3)	FlyAsh (lb/yd^3)	Coarse aggregate (lb/yd^3)	Fine aggregate (lb/yd^3)	Water (lb/yd^3)	Fly-ash (%)	Sacks of cement	w/cm
1	Lafarge	-	564	-	1850	1270	231.24	-	6	0.41
2	Lafarge	-	564	-	1850	1270	231.24	-	6	0.41
3	Lafarge	-	564	-	1850	1270	231.24	-	6	0.41
4	Lafarge	Red-rock	451.2	112.8	1850	1244	231.24	20	6	0.41
5	Lafarge	Red-rock	451.2	112.8	1850	1244	231.24	20	6	0.41
6	Lafarge	Red-rock	451.2	112.8	1850	1244	231.24	20	6	0.41
7	Holcim	Red-rock	451.2	112.8	1850	1244	231.24	20	6	0.41
8	Buzzi	Red-rock	451.2	112.8	1850	1244	231.24	20	6	0.41
9	Lafarge	Oklaunion	451.2	112.8	1850	1244	231.24	20	6	0.41
10	Lafarge	Muskogee	451.2	112.8	1850	1244	231.24	20	6	0.41
11	Lafarge	Red-Rock	488.8	122.2	1825	1175	250.5	20	6.5	0.41
12	Lafarge	Red-Rock	413.6	103.4	1880	1300	212	20	5.5	0.41
13	Lafarge	GRDA	451.2	112.8	1850	1244	231.24	20	6	0.41

Table 3.2. Fresh and Mechanical properties of Shrinkage concrete mixture

Mixture #	Cement Type	Fly-Ash Type	Shrinkage Slump (in)	Unit wt (lb/ft^3)	Air-content (%)	Compressive strength 7 Day (psi)	28 Day (psi)	Shrinkage Micro-strains 60 day (in/in)	90 day (in/in)
1	Lafarge	-	0.25	149	3.6	5237	5508	186.7E-6	296.3E-6
2	Lafarge	-	0.25	149	3.6	5458	5778	148.1E-6	296.3E-6
3	Lafarge	-	0.5	150	3.5	4788	5312	127.4E-6	281.5E-6
4	Lafarge	Red-rock	0.75	143	3.2	4750	4779	266.7E-6	367.4E-6
5	Lafarge	Red-rock	1	141	5.2	4505	4661	281.5E-6	358.5E-6
6	Lafarge	Red-rock	0.5	148	4	4192	5680	302.2E-6	376.3E-6
7	Holcim	Red-rock	0.5	144	2.5	5097	5604	284.4E-6	441.5E-6
8	Buzzi	Red-rock	0.625	144	3.1	6657	7519	317.0E-6	414.8E-6
9	Lafarge	Oklaunion	0.75	144	3.2	4720	4941	331.9E-6	432.6E-6
10	Lafarge	Muskogee	0.75	143	3	4589	5091	340.7E-6	388.1E-6
11	Lafarge	Red-Rock	1.75	141	4	4795	5472	346.7E-6	450.4E-6
12	Lafarge	Red-Rock	0.25	146	2.6	5069	5595	320.0E-6	435.6E-6
13	Lafarge	GRDA	0.625	140	3	4550	5176	275.6E-6	420.7E-6

Table 3.3. Fresh and Mechanical properties of Flexure concrete mixture

Mixture #	Cement Type	Fly-Ash Type	Flexure Slump (in)	Unit wt (lb/ft^3)	Air-content (%)	Compressive strength 3 Day (psi)	7 Day (psi)	28 Day (psi)	90 Day (psi)	Flexural strength 3 Day (psi)	7 Day (psi)	28 Day (psi)	90 Day (psi)
1	Lafarge	-	0.63	140	5.5	-	4645	4768	5987	616	624	643	739
2	Lafarge	-	-	-	-	-	-	-	-	-	-	-	-
3	Lafarge	-	-	-	-	-	-	-	-	-	-	-	-
4	Lafarge	Red-rock	1.125	138	6.125	3530	4052	5188	5762	576	590	721	767
5	Lafarge	Red-rock	1.5625	144	6.7	3589	3458	4337	5422	520	586	654	744
6	Lafarge	Red-rock	-	-	-	-	-	-	-	-	-	-	-
7	Holcim	Red-rock	0.9375	146	5.3	4032	4651	5368	5587	563	601	684	713
8	Buzzi	Red-rock	1.1875	146	6.25	3607	4275	5034	5234	588	603	716	793
9	Lafarge	Oklaunion	-	-	-	-	-	-	-	-	-	-	-
10	Lafarge	Muskogee	-	-	-	-	-	-	-	-	-	-	-
11	Lafarge	Red-Rock	-	-	-	-	-	-	-	-	-	-	-
12	Lafarge	Red-Rock	-	-	-	-	-	-	-	-	-	-	-
13	Lafarge	GRDA	1.3125	145	6.05	3514	4418	5158	5460	562	631	672	739

3.4 General mixing Procedure for flexural and shrinkage

Mixing procedure is done according to ASTM C192 with few alterations mentioned below. All of the materials for the mixtures were stored in the mixing room for at least 24 hours at 73 ^{o}F prior to mixing to keep the fresh concrete temperature constant. The mixture, shown in Table 3.1 is used having a water-to-cementitious materials ratio (w/cm) of 0.41 with 564 lb/yd^3 (equivalent to 6 sacks of cement) of total cementitious materials.

When fly ash was used in the mixture, a 20% replacement was used for the Portland cement. A dolomitic limestone from the Dolese Richard Spurr pit was used and Dolese Dover river sand for the fine aggregate. Coarse and fine aggregates were brought in from the stockpiles and individually mixed. A moisture correction for each was used to adjust the batch weights. The gravel and sand were added to the mixture first, and then 2/3 of the mixing water was added. The mixture was agitated for three minutes. Next, the fly ash, cement, and the remaining mixing water were added and mixed for three minutes. At this point the mixer was stopped and any material gathering on the sides or back of the mixer was removed over 2 minutes. After scraping, the air entrained admixture (AEA) was added, to obtain air content between 4.5 and 6.5%. After adding the AEA the mixture was agitated for three more minutes. After mixing, the slump (ASTM C143/ AASHTO T-119), unit weight or density (ASTM C138/ AASHTO T-121) and air content (ASTM C231/AASHTO T-152) was measured.

3.4.1 Shrinkage

A 2 ft^3 batch of concrete was made in a 5 ft^3 drum mixer for shrinkage mixtures. Twelve concrete cylinders were prepared out of which three were tested after 7 days and three were tested after 28 days for compression and 6 cylinders were kept for CTE test. Three concrete prisms were made for shrinkage test. These prisms were kept in lime water for 28 days and after that they were measured using a comparator. After the measurement these beams were kept in a temperature and humidity controlled environmental chamber and readings were taken every month for 64 weeks. Shrinkage specimen and shrinkage measurement using a comparator is shown in the Fig.3.2 and Fig. 3.3 respectively.

3.4.2 Flexure

A 7.5 ft^3 batch of concrete was made in a 9 ft^3 drum mixer. Twelve concrete cylinders were prepared and were tested for compression at 3,7,28 and 90 days. Also twelve beams were made for flexural test and those were tested at 3,7,28 and 90 days. Flexural test specimen and it's testing is shown in the Fig. 3.4 and Fig. 3.5 respectively. Fig 3.1 shows compression testing on concrete cylinder.

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Fig. 3.1. Crushed Concrete cylinder after the compression test.

Fig. 3.2. Picture showing shrinkage measurement on the concrete sample.

Fig. 3.3. Concrete specimen for Shrinkage test.

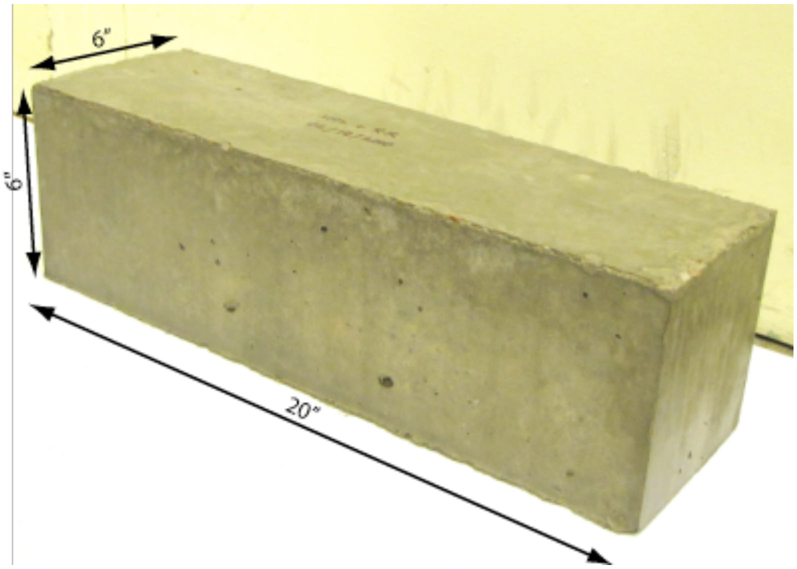

Fig. 3.4. Concrete specimen for Flexural test.

Fig. 3.5. Flexure test using a Universal testing machine

3.5 RESULTS

3.5.1 Shrinkage Results

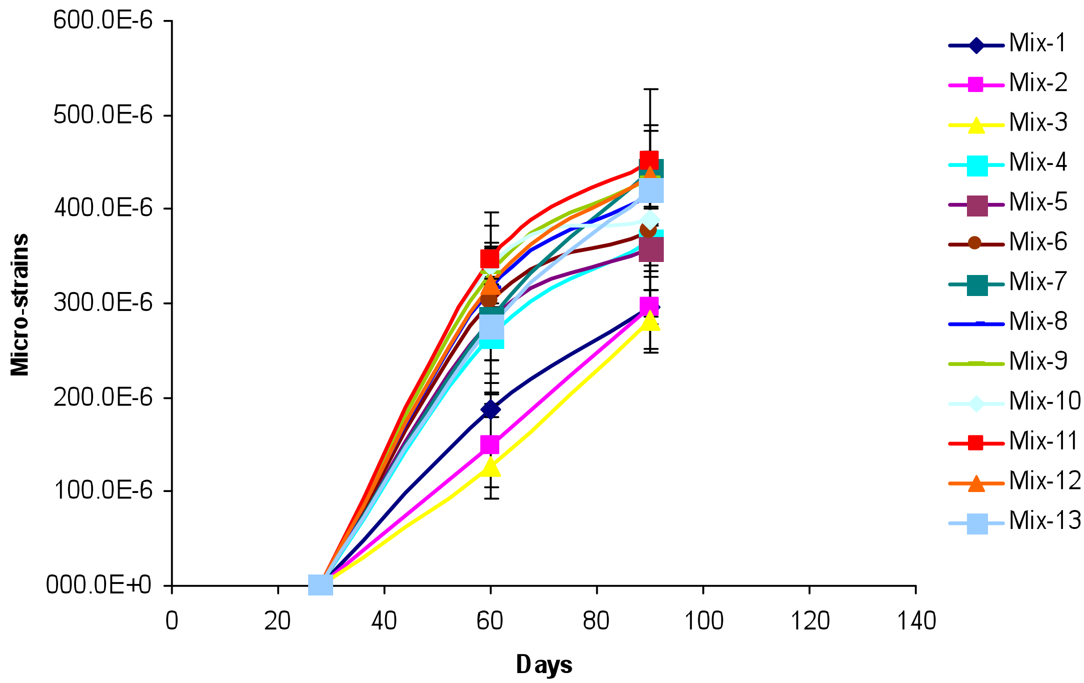

Figure 3.6- Plot of various concrete mixes showing strain with respect to days.

Shrinkage results obtained from tests conducted on various mixes are plotted above.

The Y- Axis represents strain which was obtained with respect to 28 day shrinkage value.

The X-axis represents days on which shrinkage is checked.

3.5.2 Compressive strength results for shrinkage mixture

Compressive strength results for shrinkage are listed in Table 3.2

3.5.3 Flexure Results

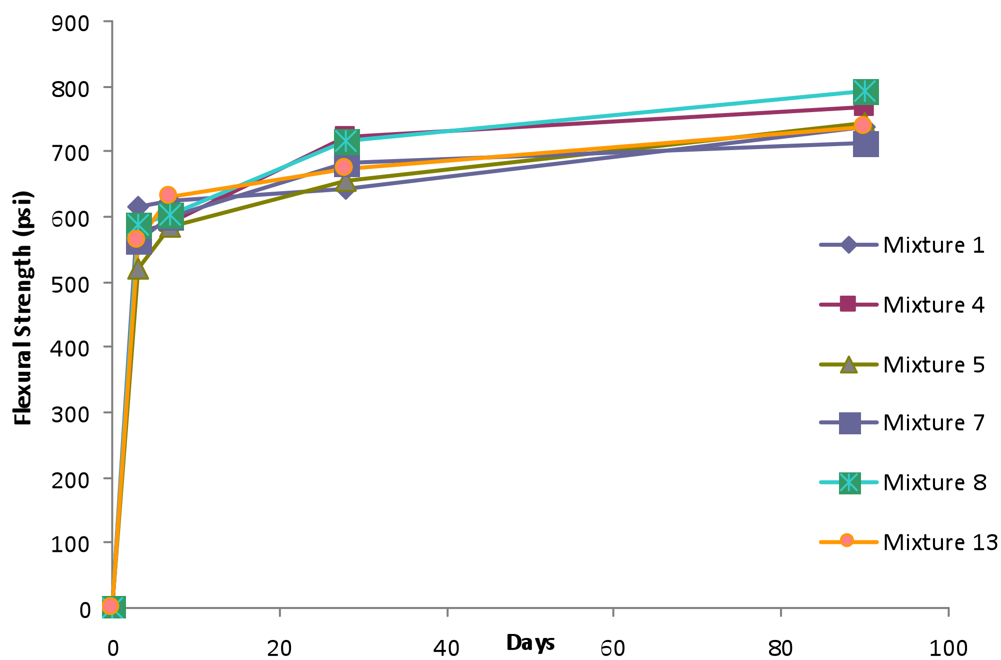

Figure 3.7- Plot of various concrete mixes showing Flexural strength (psi) with respect to days

Flexural strength results obtained from test conducted on various mixtures is shown in Fig. 3.7. The Y- axis represents flexural strength at 3, 7, 28 days. The X-axis represents days on which flexure sample is tested.

3.5.4 Compressive strength results for Flexural mixtures

Compressive strength results for shrinkage are listed in Table 3.3

3.5.5 Comparison between flexural and compressive mixtures

A comparison between the flexural and compressive strengths and the $9.5\sqrt{f'c}$ line is shown in Fig. 3.8.

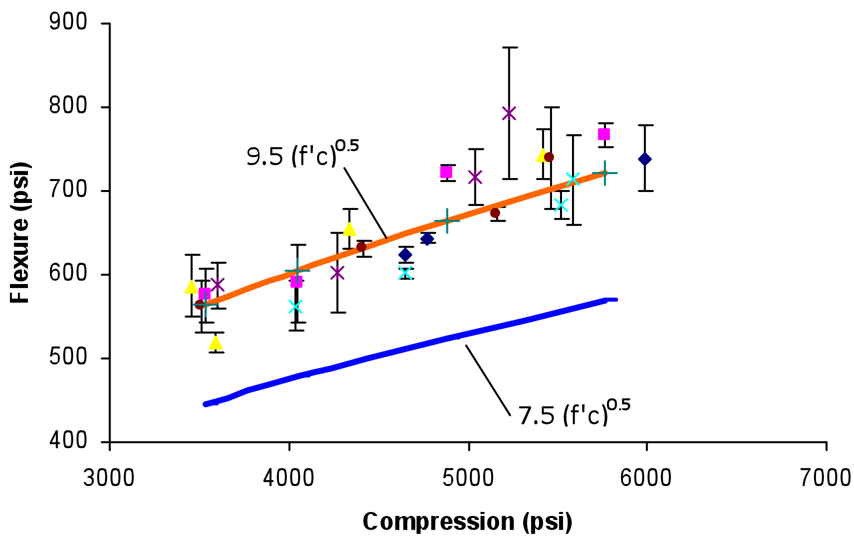

Figure 3.8- Plot of various concrete mixes showing Flexural stress (psi) with respect to compressive strength (psi).

3.6 DISCUSSION

3.6.1 Discussion on Shrinkage

From the data obtained it can be understood that different mixes have different intensities of shrinkage but the trend of the behavior is same which can be seen in the Fig 3.6. Graph plotted in Fig.3.6 represents the typical PCC behavior on drying. Total 13 mixes were made to check the shrinkage.Mixture-11 which is 6.5 sacks of cement mix, consisting of Lafarge cement and fly-ash from Red-rock showed maximum shrinkage. As 6.5 sacks of cement mixture contains more cementitious material, higher shrinkage was expected. Mixture-3 which is a straight cement concrete mixture made with Lafarge cement showed minimum shrinkage. Additional readings are being taken on the specimens and will be reported in future test results.

3.6.2 Discussion on Compressive strength for shrinkage mixture

All the specimens were above 4200 psi at 28 day. Mixture-8 showed maximum compressive strength for 7 and 28 day test. Mixture-8 contains Buzzi cement and fly-ash from Red rock. Mixture-6 showed minimum compressive strength for 7 day test while Mixture-5 showed minimum compressive strength for 28 day test. Both Mixture-5 and Mixture-6 contain Lafarge cement and fly-ash from Red-rock.

3.6.3 Discussion on Flexural strength

Engineers use flexural strength to design pavements and so these measurements can be useful. From the above Fig.3.7 it can be said that all the beams are following the same curve. Mixture-1 shows maximum flexural strength at 3 day test. Mixture-1 is a straight cement concrete mix made of Lafarge cement. Mixture-13 showed maximum strength at 7 day test. Mixture -13 is made up of Lafarge cement and fly- ash from GRDA. Mixture-4 had maximum strength at 28 day test. Mixture-4 is made up with Lafarge cement and fly-ash from Red rock. Mixture-8 had maximum strength at 90 day test. Mixture-4 is a mix made up with Buzzi cement and fly-ash from Red rock. Where as Mixture-5 showed minimum strength at 3 and 7 day test. Mix-5 consists of Lafarge cement and fly-ash from Red-rock. Mixture-1 showed minimum strength at 28 day test. Mixture -1 is a straight cement concrete mix made with Lafarge cement. Mixture-7 showed minimum strength at 90 day test. Mix-7 consists of Holcim cement and fly-ash from Red-rock.

3.6.4 Discussion on Compressive strength for Flexural strength mix

From the Table.3.3 it can be said that Mixure-7 is having maximum compressive strength at 3, 7 and 28 day test. Mixture-7 is made up of Holcim cement and fly-ash from Red-rock. . Mixture -1 showed maximum strength at 90 day test. Mixture-1 is straight concrete cement mix. Mixture -13 showed minimum strength at 3 day test. Mixture-13 consisted of GRDA fly-ash and Lafarge cement. Mixture-5 showed minimum strength for 7 and 28 day test. Mixture-5 consisted of Lafarge cement and fly-ash from Red-rock. Mixture -8 showed minimum strength at 90 day test. Mixture-8 consisted of Buzzi cement and fly-ash from Red-rock.

3.6.5 Discussion on relationship between Flexural stress and Compressive stress

Graph in fig 3.8 shows the relationship between flexure and compressive stress for a concrete specimen. MEPDG considers Modulus of rupture i.e. flexural stress to be $9.5\sqrt{f'c}$. At the start, graph was drawn between compressive stress and the actual flexural stress calculated by testing the beams in bending. This graph was compared with compressive stress and flexural stress calculated using the above equation. Looking at both the graphs we can say that both the graphs are very close to each other.

3.7 Conclusion on shrinkage and Flexure results

Shrinkage tests which were conducted on 13 typical Oklahoma pavement mixes showed positive results. For most of the Shrinkage mixtures, the strain values which were noted on the 60th day were in the acceptable or typical shrinkage range. According ACI 209.1R-05 the long term concrete shrinkage is about 200 to 800 micro strains. Shrinkage test requires a considerable amount of time to reach the end result. According to ASTM-C157 it takes about 64 weeks for a concrete specimen to reach the ultimate shrinkage value. Test is still in process and is performed on each specimen every month.

Six concrete beams were tested for flexure; all beams were made using typical Oklahoma pavement mix. Flexure results can be verified using the relation between Flexural stress i.e. Modulus of rupture with compressive stress of the concrete, MR = $k\sqrt{f'c}$ .For inch-pound units, value of "k" varies from 9 to 11 depending on the types of aggregates. According to MEPDG design guide, "k" is taken as 9.5. Majority of the flexure specimen have the "k" value close to 9.5, which is acceptable. Looking at Flexure test graphs, it can be concluded that the specimen tested showed typical concrete beam behavior under bending.

CHAPTER IV

CONCLUSIONS

As ODOT wanted to implement MEPDG for the Oklahoma pavements, the first and the important step to start with was to understand MEPDG and its sensitive parameters. Various sensitive parameters were tested and there impact on the pavements in terms of thickness change was noted. Technique and the method used to understand the impact of the various sensitive variables in chapter 2 “Sensitive analysis of the rigid pavement design with the Mechanistic empirical pavement design guide” is not found in any previous publication or journal paper.   In second part the important parameters in MEPDG like Shrinkage, flexure and compression were tested on common concrete mixtures from the state of Oklahoma. This was done to improve the input parameters in MEPDG design.

4.1 Sensitivity analysis

From sensitivity analysis it was seen that several variables made much more impact than expected and also several variables behaved in ways that were unexpected. Sensitivity analysis allowed the user to quantitatively compare the impact of different variables on the design thickness in the MEPDG for CRCP and JPCP.

No combinations of the variables were tried and tested. Also the owner should be careful in using the MEPDG for pavement design as the results of the analysis do not seem intuitive as several small changes in input data required pavement thickness designs of significantly different values. Furthermore, the equations and processes used to calculate the performance of the pavements should be more transparent and easier to understand to the user of the software.

4.2 Shrinkage and Flexural test

Shrinkage mixtures that were tested had the strain values noted on the 90th day under the acceptable or typical shrinkage range. Shrinkage tests are still in process and the final ultimate shrinkage results are expected after 64 weeks.

For flexure, relation between Flexural stress and compressive stress were tested. Majority of the specimen had the “k” value close to 9.5, which is acceptable. As according to MEPDG, Modulus of rupture (MR) $= 9.5\sqrt{f'c}$. Also Looking at Flexure test graphs, it can be concluded that the specimen tested showed typical concrete beam behavior under bending.

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List of ASTM and AASHTO test methods

ASTM C157 / C157M - 04 Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete.

ASTM C78 / C78M - 08 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

ASTM C39 / C39M - 05 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C143 / C143M - 08 Standard Test Method for Slump of Hydraulic-Cement Concrete

ASTM C138 / C138M - 08 Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete

ASTM C231 / C231M - 08c Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method

AASHTO T 160 Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete

AASHTO T 97 Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

AASHTO T 22 Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens

AASHTO T119 Standard Specification for Slump of Hydraulic Cement Concrete

AASHTO T 121 Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete

AASHTO T 152 Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method

VITA

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Master of Science

Thesis: INVESTIGATION OF THE MEPDG FOR USE IN OKLAHOMA CONCRETE PAVEMENTS

Major Field: Civil Engineering

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Shardul Kadam was born in Mumbai, Maharashtra, India on March 18, 1984. His parents are Subhash.J.Kadam and Sunetra.S.Kadam.

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Title of Study: INVESTIGATION OF THE MEPDG FOR USE IN OKLAHOMA CONCRETE PAVEMENTS

Pages in Study: 44 Candidate for the Degree of Master of Science

Major Field: Civil Engineering

Scope and Method of Study:

The Oklahoma Department of Transportation (ODOT) wants to implement the Mechanistic-Empirical Pavement Design Guide (MEPDG) to design pavements. But before implementing the MEPGD it is necessary to understand which design parameters are the most sensitive. The Impact of these parameters in terms of change of pavement thickness was carried out. Further to improve the input parameters in MEPDG, it was decided to obtain actual shrinkage and strength values for typical Oklahoma pavement mixtures.

Findings and Conclusions:

From sensitivity analysis it was seen that several variables made much more impact than expected and also several variables behaved in ways that were unexpected. Furthermore, the equations and the processes used to calculate the performance of the pavements should be more transparent and easier to understand to the users of the software. Shrinkage mixtures that were tested are under the acceptable or typical shrinkage range. Shrinkage testing is still in progress and the final ultimate shrinkage results are expected after 64 weeks. For flexural strength, relationship between flexural stress and compressive stress i.e. M.R = $k\sqrt{f'c}$ was checked. The majority of specimens had the “k” value close to 9.5, which is acceptable according to MEPDG. Also all the flexural stress graphs showed typical concrete beam behavior under bending and reasonable strengths.